SRR-86-08

Load Testing and Analysis of Wood Bridges Rehabilitated by Transverse Post-Tensioning

Ministry of Transportation and Communications
Ontario

Research and Development Branch

86-05322/

Technical Report Documentation Page

MTC Report Number SRR-86-08	ISBN Number:	Other Document Number:	Date: July 1986

Title: Load Testing and Analysis of Wood Bridges Rehabilitated by Transverse Post-Tensioning

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Published by:
The Research and Development Branch
Ministry of Transportation and Communications

Client Group:

Participating Agencies:

Abstract:
The method of transverse post-tensioning laminated wood bridges was conceived in 1976 as a means of rehabilitating existing nailed laminated decks. Since that time the concept has undergone considerable research and development. Specifications for the design of new bridges and the rehabilitation of existing ones have been produced and were adopted by the Ontario Highway Bridge Design Code.

This report describes the load testing and evaluation of two MTC bridges which were rehabilitated by transverse post-tensioning. These bridges had not undergone the required re-post-tensioning subsequent to rehabilitation and so exhibited high post-tensioning losses. The objective of the tests was to evaluate the effects of the very low post-tensioning levels on the structural performance of the bridges.

Both bridges were load tested before and after re-post-tensioning. Even under low levels of post-tensioning, the laminated decks did not display any signs of distress under the fully loaded MTC test vehicles (2 1/2 times the legal load). In addition, the structural response of these decks was found to be representative of an orthotropic plate.

Key Words: post-tensioning, wood bridges, laminated bridge decks, OHBDC

Comments:

Distribution: MTC Head Office and Regions, selected structural engineers

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SRR-86-08

Load Testing and Analysis of Wood Bridges Rehabilitated by Transverse Post-Tensioning

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Published by
The Research and Development Branch
Ontario Ministry of Transportation and Communications
Honourable Ed Fulton, Minister
D.G. Hobbs, Deputy Minister

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July 1986

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ACKNOWLEDGEMENTS

The following M.T.C. employees and their support staff are given specific acknowledgement for their valued contributions and overall support which enabled the successful completion of this research project.

J. Klubal, Senior Research Technician, Downsview
W. Kulmatickus, Head, Structural Office, Thunder Bay (Retired)
W. Matyczuk, Senior Services Supervisor, Thunder Bay
M. Hanson, A. Messervia and Thunder Bay District Bridge Crew
W. Kwok, Supervisor, Instrumentation

1/ INTRODUCTION

The method of transverse post-tensioning was first conceived in 1976 as a means of rehabilitating existing nailed laminated decks [1]. Its success as a method of rehabilitation resulted in it becoming the subject of an extensive research and development program [2]. This program resulted in the development of a set of design specifications for post-tensioned or *prestressed wood, covering both new and existing bridges. These specifications, which have since been adopted by the Ontario Highway Bridge Design Code (OHBDC) [3], were used to design a number of wood bridges. Some of these bridges were to be used as test sites to generate additional information on their structural performance.

This report documents the 1982 load testing and subsequent evaluation of the Kabaigon River and Pickerel River Bridges on Highway 11, in the district of Thunder Bay. These bridges were rehabilitated by transverse post-tensioning in 1980 and 1981 respectively, and had not undergone the required re-post-tensioning in accordance with the design requirements and so exhibited high losses.

The objective of the project was two-fold. First to perform a proof load test on the decks to evaluate their strength relative to the very low levels of post-tensioning. Second, to evaluate the structural performance of the decks due to changes in the post-tensioning forces. The latter was to be performed by load testing the bridges before and after the re-post-tensioning of the decks.

This report deals primarily with the results from the Kabaigon River Bridge test as the Pickerel River test produced very similar results.

*Prestressed wood is the accepted terminology for transversely post-tensioned wood decks as adopted by the OHBDC [3].

2/ SITE DESCRIPTION

The Kabaigon River Bridge MTC Site No. 48 W-19 is a two lane, multi-span structure located on Highway 11 about 12.9 km west of secondary Highway 586, in the district of Thunder Bay, Ontario. The 5-span continuous structure was originally designed and built for an H20 truck load in 1951 and later rehabilitated by transverse post-tensioning in September 1980.

The 22.86 m (75 ft.) long structure consists of two side spans of 4.11 m (13.5 ft.) and three intermediate spans of 4.88 m (16 ft.) as shown in Figure 1. The 63 mm x 254 mm (2 1/2 in. x 10 in. actual) wood laminations were placed on edge and oriented longitudinally, as illustrated in Figure 2, and nailed every 305 mm (12 in.) with 114 mm (4 1/2 in.) nails. The laminations were so arranged to produce longitudinal discontinuities of the laminates at the one-third span points and supports only. In addition, for any one transverse section, the joints would occur in every third lamination.

All wood was pressure treated with creosote oil prior to use and an asphalt wearing surface was applied directly on the timber deck. The Pickerel River Bridge MTC Site No. 45-96 is also on Highway 11 and is situated about 9.8 km east of Highway 633 near Atikokan. Except for supporting an overall bridge length of 23.2 m it is identical to the Kabaigon River Bridge.

3/ INSTRUMENTATION

The instrumentation layout, as shown in Figure 3 was the same for both bridges and consisted of 47 linear displacement transducers. Twenty-six were used to measure the vertical displacement contours across the centre of the east end span (Span No. 1) and the adjacent intermediate span (Span No. 2) on both bridges. The remaining transducers were used to monitor the vertical movement of the pile bents supporting the same two spans.

The accuracy of these linear displacement transducers, considering the installation procedure, is believed to be within 0.14 mm. The instruments were monitored using a 100 channel VIDAR Autodata Scanning System.

4/ LOAD TESTING AND ANALYSIS

The following is a discussion of the load testing results dealing primarily with the Kabaigon River Bridge. The Pickerel River Bridge produced nearly identical results so only limited data is presented.

4.1/ Test Procedure

Load testing of the Kabaigon River Bridge was performed in September, 1982, using two of MTC's load testing vehicles which are shown in Figure 4.

The vehicles were loaded incrementally to produce the six vehicle weights given in Table 1. This incremental loading procedure enables the early detection of non-linear performance of the structure which is indicative of structural distress. At each load level, the vehicle was moved through the vehicle positions shown in Figure 5. All displacement transducers were recorded for each stationary position of the vehicle.

Both bridges were load tested before and after the deck was re-post-tensioned. After re-post-tensioning, only load cases 3, 7 and 8 were performed in order to produce only enough data for comparison purposes. Both bridges supported the fully loaded test vehicles without any sign of distress.

It should be noted that neither bridge was load tested prior to initial rehabilitation. Such testing was not considered practicable as these delaminated nailed deck systems display non-linear mechanical performance [1].

The levels of prestress pressure before and after re-post-tensioning are given in Table 2. The minimum design valve is 350 kPa, therefore the Pickerel River Bridge provided an opportunity to proof-test a structure below the minimum.

4.2/ Test Results

The bridges were load tested, before and after re-post-tensioning, up to the maximum load shown in Table 1. No signs of structural deficiency were observed.

Figures 6 through 11 compare some of the midspan deflections under increasing loads measured at the Kabaigon River Bridge. These displacements have been adjusted to account for measured support movements as described in Figure 12. Figures 6 and 7 display the displacements before re-post-tensioning and Figures 8 and 9 after. Comparing these displacements, it can be seen that re-post-tensioning reduced the maximum measured deflections. Figures 10 and 11 compare the midspan deflections from both conditions under load level 8. These plots indicate that the increase in the transverse post-tensioning force has only marginally changed the distribution characteristics of the bridge while reducing the overall bridge deflections. This apparent stiffening of the bridge was also observed at Pickerel River Bridge as shown in Figures 13 and 14 and agrees with previous load testing data [1].

The displacements given in Figure 13 and 14 for the Pickerel River Bridge must be evaluated with caution. The method of compensating for pile movements as described is Figure 12 requires that the test vehicles be aligned parallel to the roadway. Site conditions and time constraints made this difficult. In many instances the two tandems of the test vehicle were not properly in line with each other. For this reason, the subsequent evaluation concentrated on the results from the Kabaigon River Bridge test.

4.3/ Analysis

The current OHBDC [3] includes design specification for prestressed wood bridges. The analysis section of the Code assumes these decks perform as orthotropic plates based upon extensive research and testing [1,2].

In order to verify the analytical design assumptions, the Kabaigon River Bridge was analysed using a program called ORTHOP [4]. This program simulates the deck as a plate which has flexural and torsional properties that are different in the two principal directions. The material property assumptions were taken from the OHBDC which provided a longitudinal modulus of elasticity, E, of 9500 MPa. The effective transverse modulus of elasticity was assumed to be .05 E. The effective shear modulus was assumed to be 0.065 E. The idealization of the deck conformed to the requirements specified in the analysis section of the OHBDC.

The ORTHOP program allows the input of wheel loads as patches so the specified test vehicle placement could be simulated quite readily. However, in discussing the subsequent analytical comparisons, it is important to recognize that during field testing the accuracy achieved in vehicle placement is such that the placement may vary up to 200 mm in any direction.

Some of the analysis results are compared to the measured displacements in Figures 10 and 11. The modulus of elasticity of wood generally has a coefficient of variation of over 25%, therefore, it is unlikely that the assumed modulus would closely match the actual stiffness of the bridge. This is demonstrated in Figures 10 and 11 where the predicted displacements are larger than those measured. The shapes of the deflection curves, which are indicative of distribution characteristics of the deck, can be compared. In both Figures it appears the orthotropic analysis produced deflection curves closer in shape to the results derived from the initial testing. Here the transverse stiffness of the deck, which is directly affected by the level of post-tensioning, was lower.

Figures 15 and 16 display the same deflection comparisons where the curves have been normalized in order to offset the effects of error in the assumed modulus of elasticity. In addition, Figures 17 and 18 compare normalized contours for situations of a single vehicle. In all instances, the assumed orthotropic analysis appears to better represent the initial testing condition, prior to re-post-tensioning.

4.4/ Summary

The load test results confirmed the applicability of the orthotropic analysis to represent a transversely post-tensioned wood deck under low levels of post-tensioning. This is desirable since the design process should account for the worst situation.

There is an improvement in the distribution characteristics as the level of post-tensioning increases. However, the magnitude of these changes is not significant enough to affect the strength in the lower range of post-tensioning assumed in design.

5/ EVALUATION

The structures were evaluated using the OHBDC specifications which provided a factored moment resistance of 164 kN-m/m width assuming Jack Pine No. 2. This assumes a load sharing factor of 1.5 as applied to prestressed wood decks. Assuming a nailed laminated deck, as was the case prior to the initial rehabilitation the factored moment resistance would have been 131 kN-m.

Using the simplified methods of analysis specified in the OHBDC, a design distribution width (D_d) of 1.60 m was established. This provided a factored load effect at the ultimate limit state of 125 kN-m/m width. Therefore, the rehabilitated structure has considerable reserve strength. For nailed laminated decks the specified distribution width is 0.85 m which results in a factored load effect of 221 kN-m/m width. Therefore, prior to rehabilitation, the structure was significantly over-stressed according to the OHBDC.

Utilizing the midspan deflection results from the Kabaigon River Bridge a distribution width of 1.95 m was estimated. Though the distribution for deflections is always better than that for moments the much larger value of D_d = 1.95 m is indicative of a better distribution than that given by the OHBDC. The possible discrepancy arises from the relatively short spans of the bridge which result in structural parameters that do not reside within the scope of the given distribution curves in the OHBDC. The previous D_d value of 1.6 m was estimated based upon extrapolation of the given curves.

Further evaluation was performed under the SLS Type 1 loading of the OHBDC. The OHBDC limits the maximum deflection under a single vehicle to L/400 which corresponds to 12 mm for this structure. The calculated deflection was found to be 7 mm. Under actual testing using loading similar to the SLS Type 1, the maximum measured deflection was less than 5 mm.

6/ CONCLUSIONS

Increasing the transverse post-tensioning forces definitely improves the load distribution characteristics of the deck. In the case of the Kabaigon River Bridge, the peak deflection under load Level 8 was reduced by about 20 to 30%. However, this occurred due to a change in prestress pressure from 510 kPa to 1140 kPa which is over a 100% increase.

In design, according to the OHBDC specifications, the final prestress pressure can range from no less than 350 kPa to no more than 590 kPa. The latter was determined for Jack Pine. However, only Jack Pine and Douglas Fir can achieve this maximum level: the majority of species would not exceed 410 kPa. Therefore, considering the narrow range of final prestress levels it is considered realistic to ignore the effects of changes in the level as it might affect the distribution characteristics used in design.

The overall effects of rehabilitating these bridges by transverse post-tensioning, was to increase their live load carrying capacity by a factor of more than two. The member resistance is increased by 25% due to the increased load sharing performance, while the live load effects are reduced by about 45% due to improved load distribution.

8/ REFERENCES

[1] Taylor, R.J., and Csagoly, P.F., "Transverse Post-Tensioning of Longitudinally Laminated Timber Bridge Decks," RR 220, Ontario Ministry of Transportation and Communications [MTC], Downsview, Ontario, June 1979.

[2] Taylor, R.J., Batchelor, B. DeV. and Van Dalen, K., "Prestressed Wood Bridges," Research Report SRR-83-01, MTC, Downsview, Ontario, April 1983.

[3] Ontario Highway Bridge Design Code, Ontario Ministry of Transportation and Communications, Downsview, Ontario, 1983.

[4] Bakht, B. and Bullen, R.C., "Analysis of Orthotropic Right Bridge Decks," Highway Engineering Computer Branch B/15 (ORTHOP), Department of Environment, London, England, 1973.

Table 1/ Tandem Weights of Vehicle Testing

Test Vehicle No. 2 (MACK) Loading

Load Level	Number of Blocks	Weight Steering Tandem kg	Weight Front Tandem kg	Weight Rear Tandem
1	22	4 700	14 500	22 700
3	38	5 200	20 000	32 200
5	50	5 700	24 500	38 500
6	54	5 800	25 900	40 800
7	56	5 800	25 900	43 100
8	60	5 900	26 800	45 800

1 kg = 2.205 lb.

1 kg = 9.806 N

Table 2/ Levels of Prestress Pressure

	DATE OF REHABILITATION	LEVEL OF PRESTRESS PRESSURE (kPa)				
		SEPT, 1980	JUNE 1981	JULY 1982	SEPT 1982*	OCT 1982*
KABAIGON R.	SEPT. 1980	1140	—	510	510	—
PICKEREL R.	JUNE 1981	—	1090	255	—	255

* Date of testing respective bridges. Both bridge re-post-tensioned to 1100 MPa at these times.

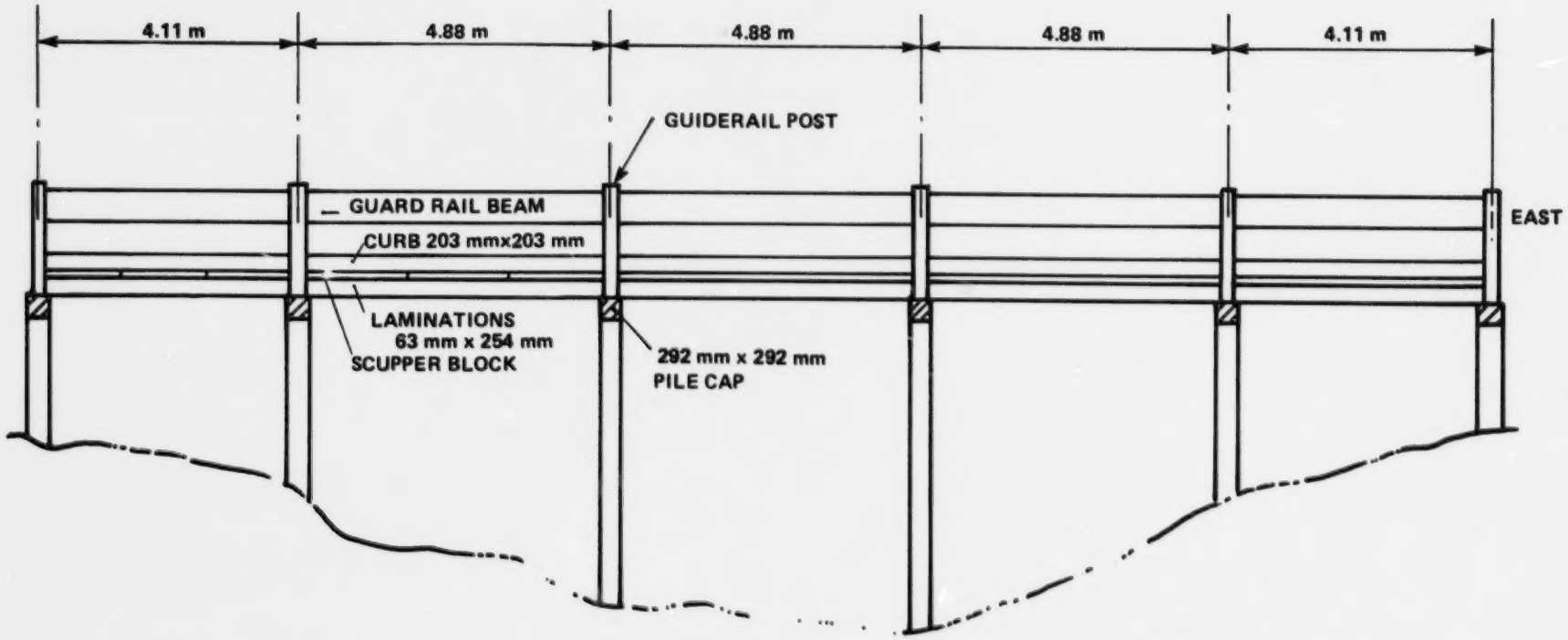


Figure 1/ Highway 11, Elevation of Kabalgon River Bridge

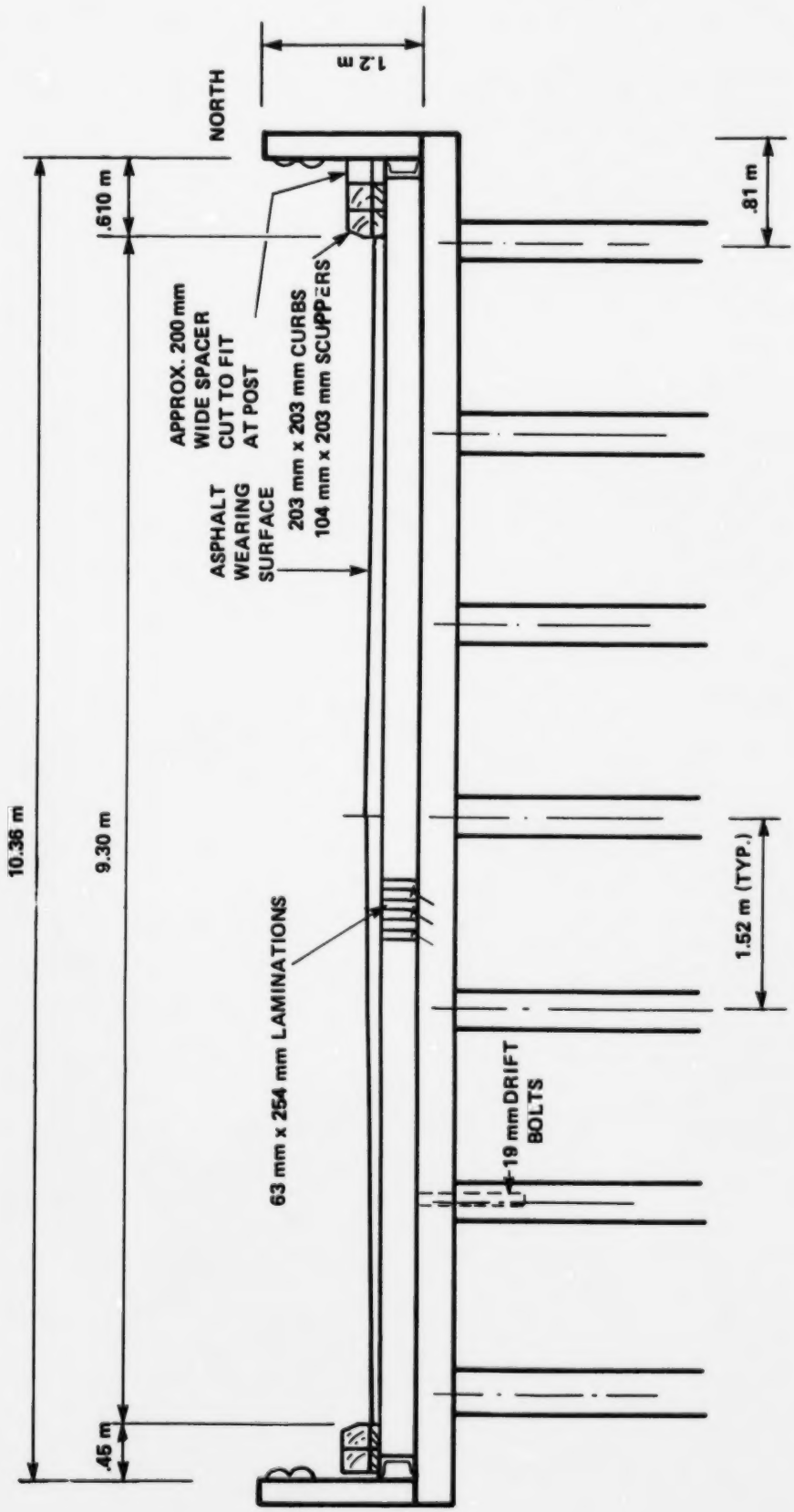


Figure 2/ Cross-section of Deck at Pier, Kabaigon River Bridge

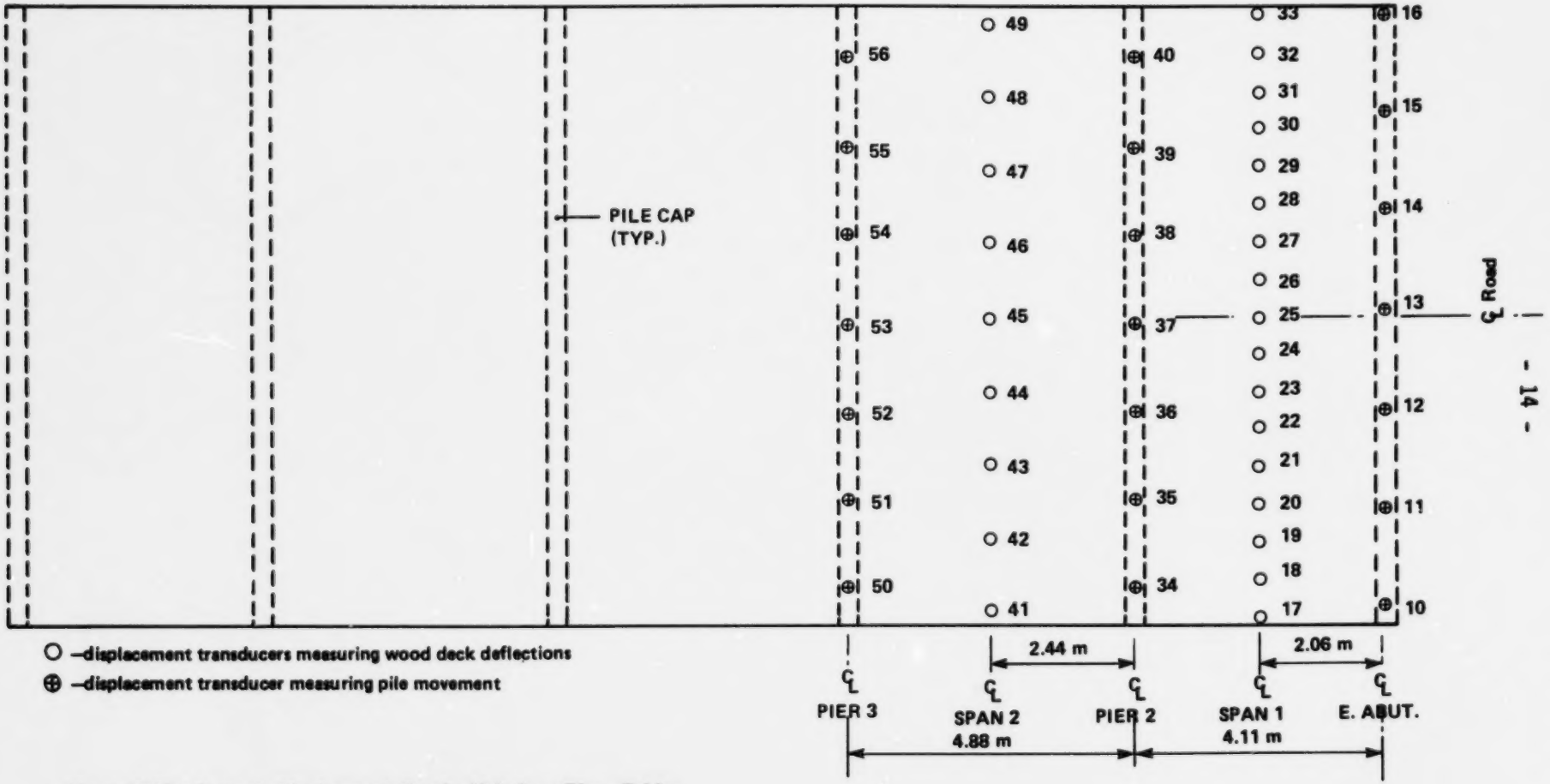


Figure 3/ Plan Layout of Instrumentation for Kabalgon River Bridge

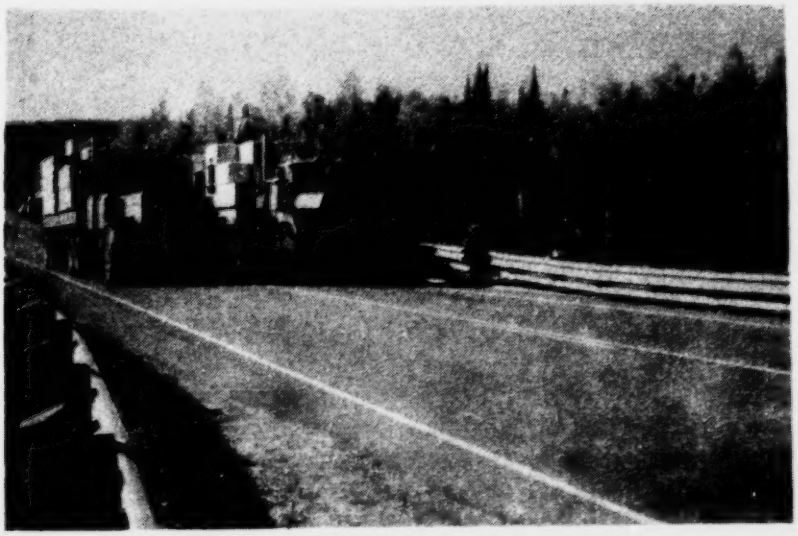
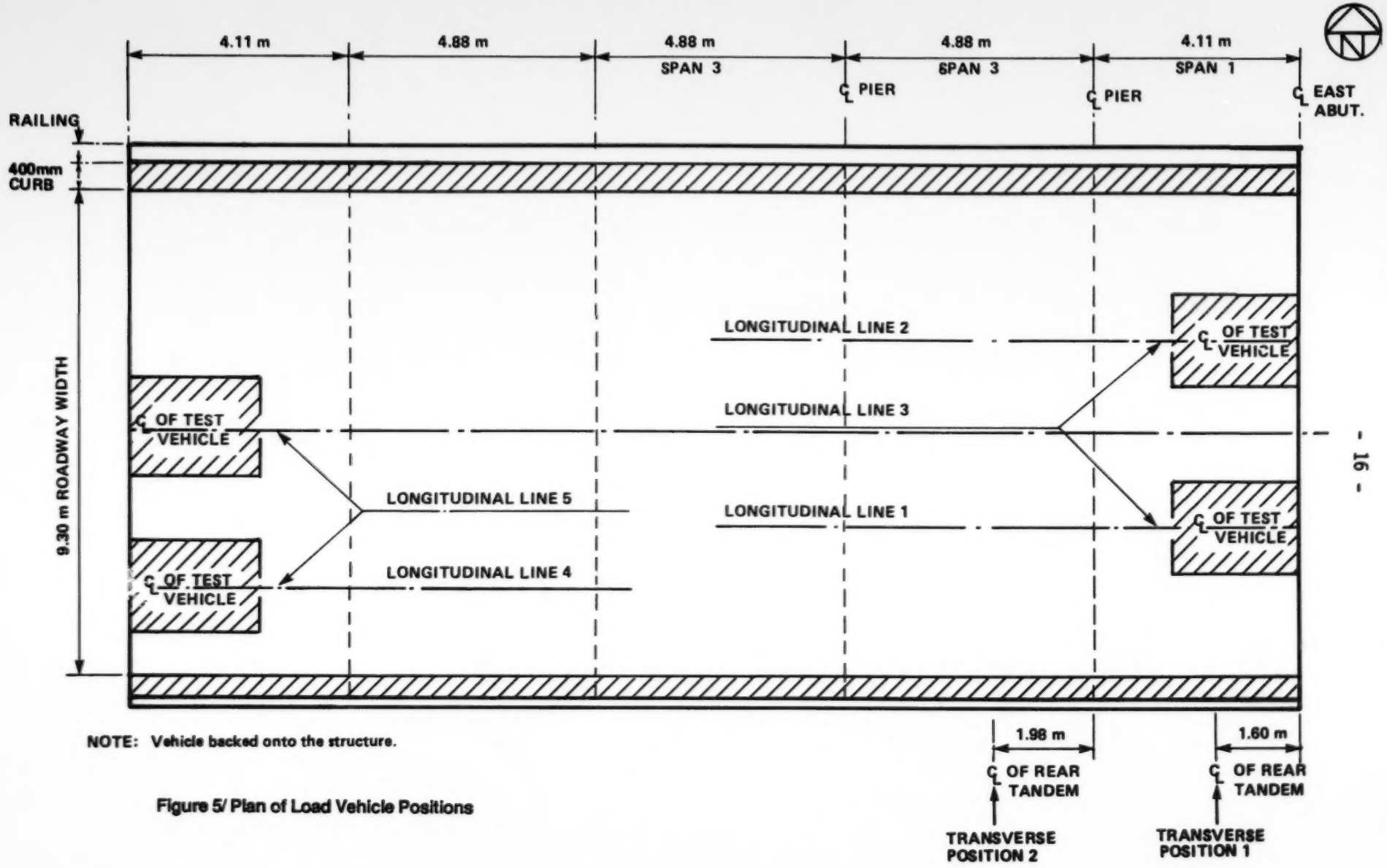


Figure 4/ Load Testing Vehicles



NOTE: Vehicle backed onto the structure.

Figure 5/ Plan of Load Vehicle Positions

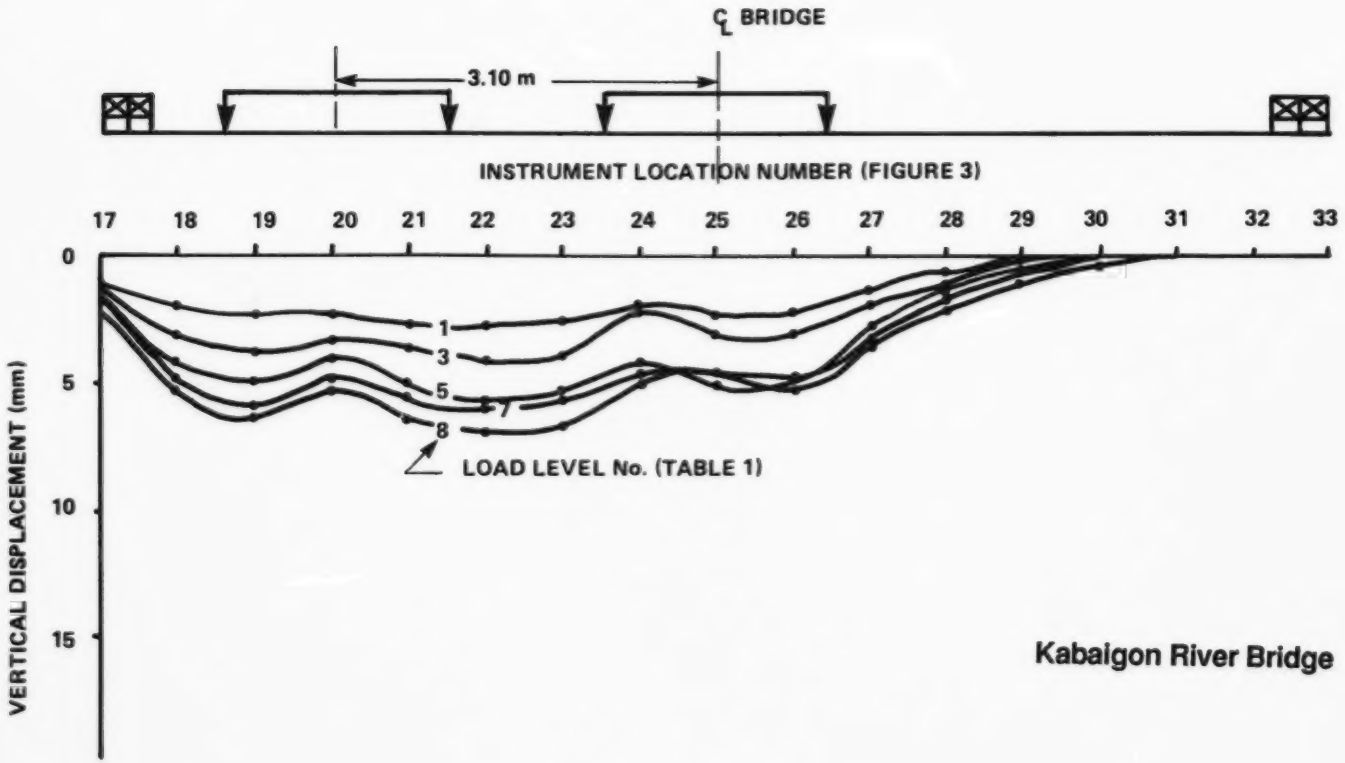


Figure 6/ Midspan Deflections, Span No. 1, Before Re-post-tensioning

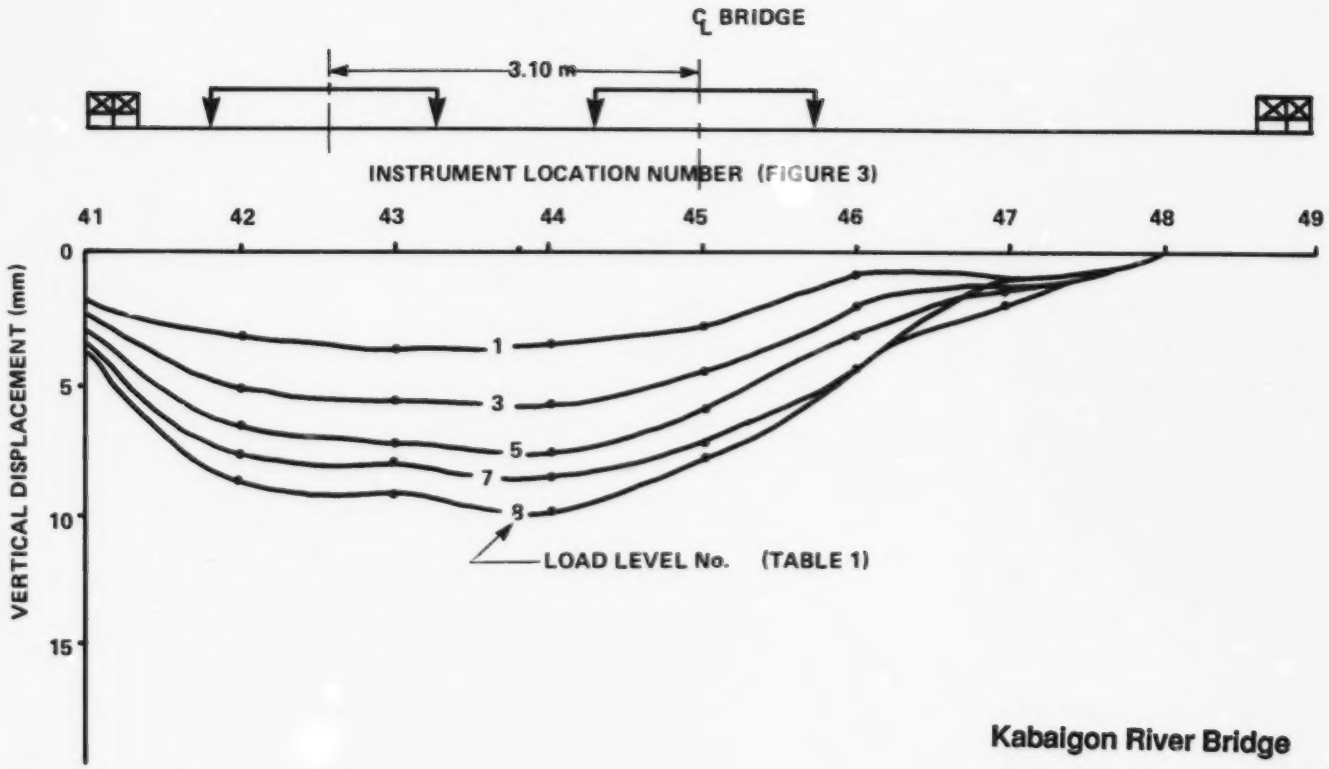


Figure 7/ Midspan Deflections, Span No. 2, Before Re-post-tensioning

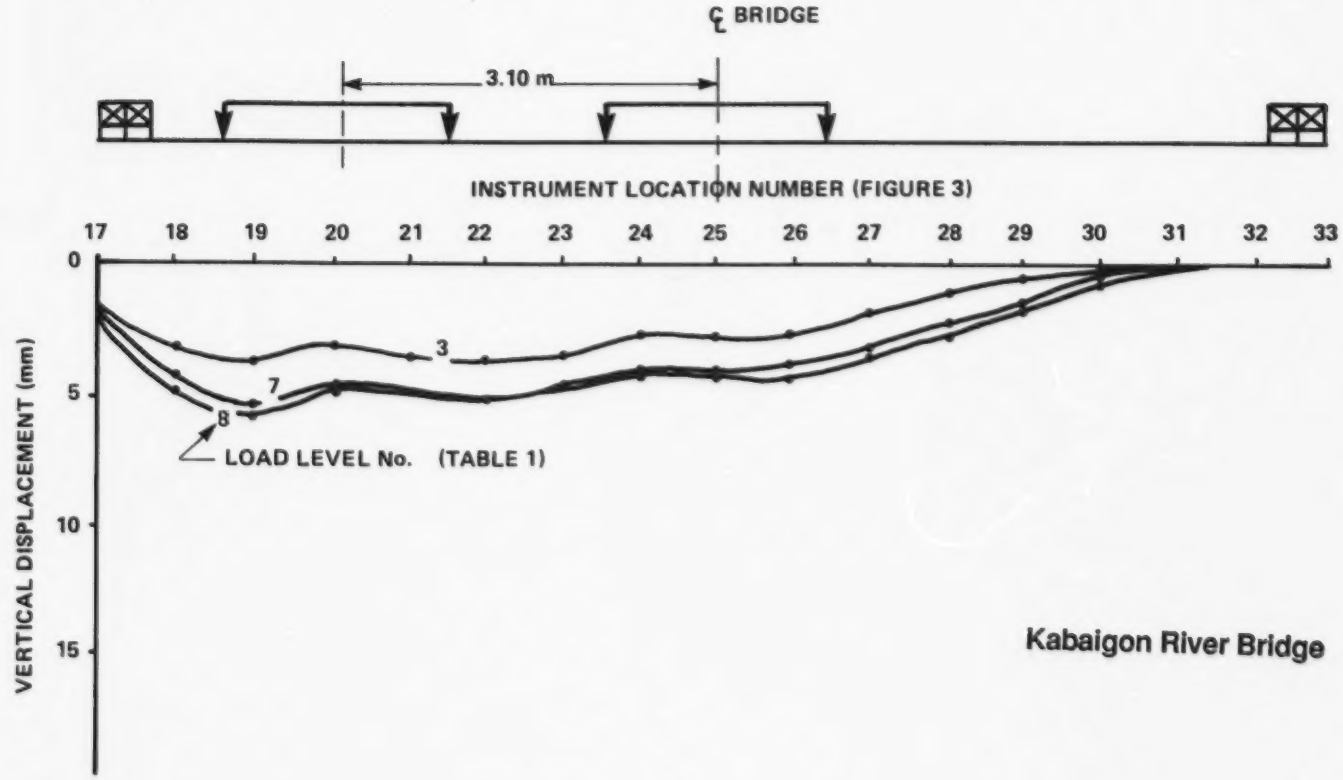


Figure 8/ Midspan Deflections, Span No. 1, After Re-post-tensioning

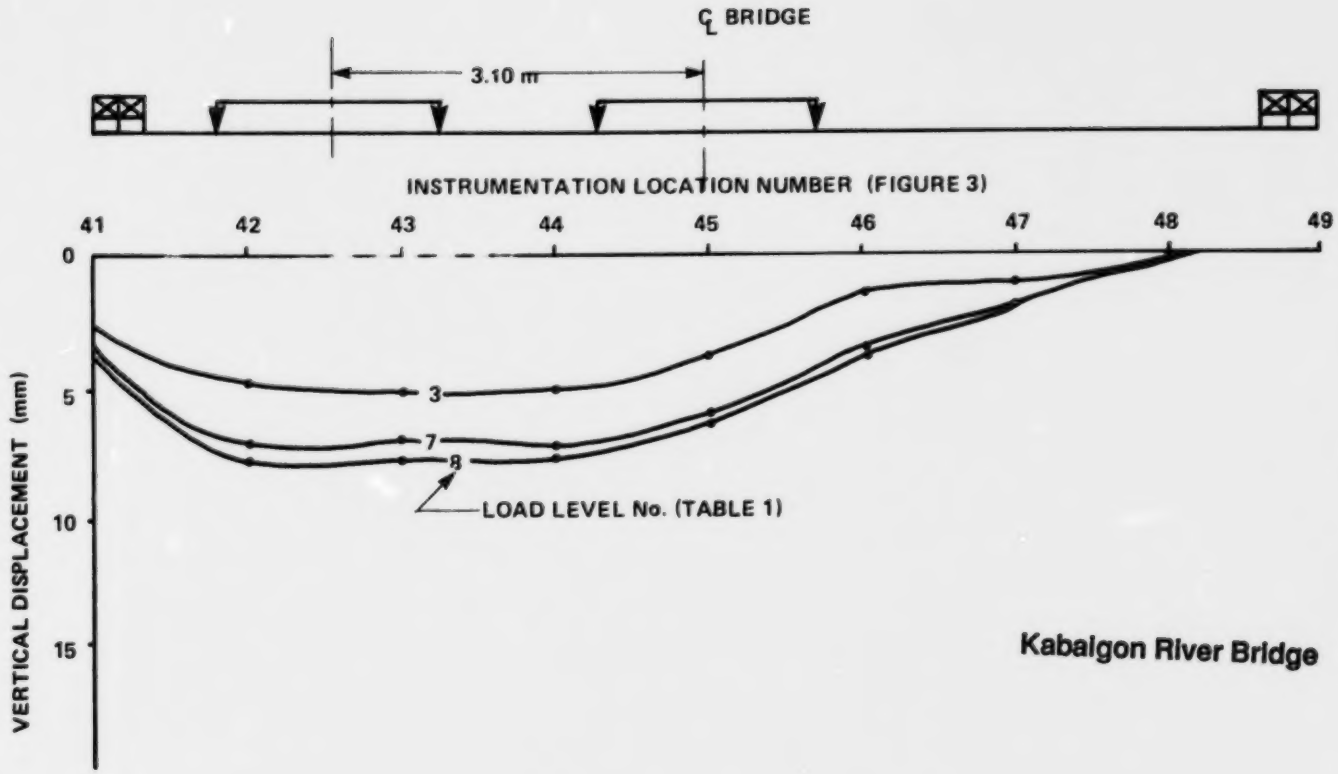


Figure 9/ Midspan Deflections, Span No. 2, After Re-post-tensioning

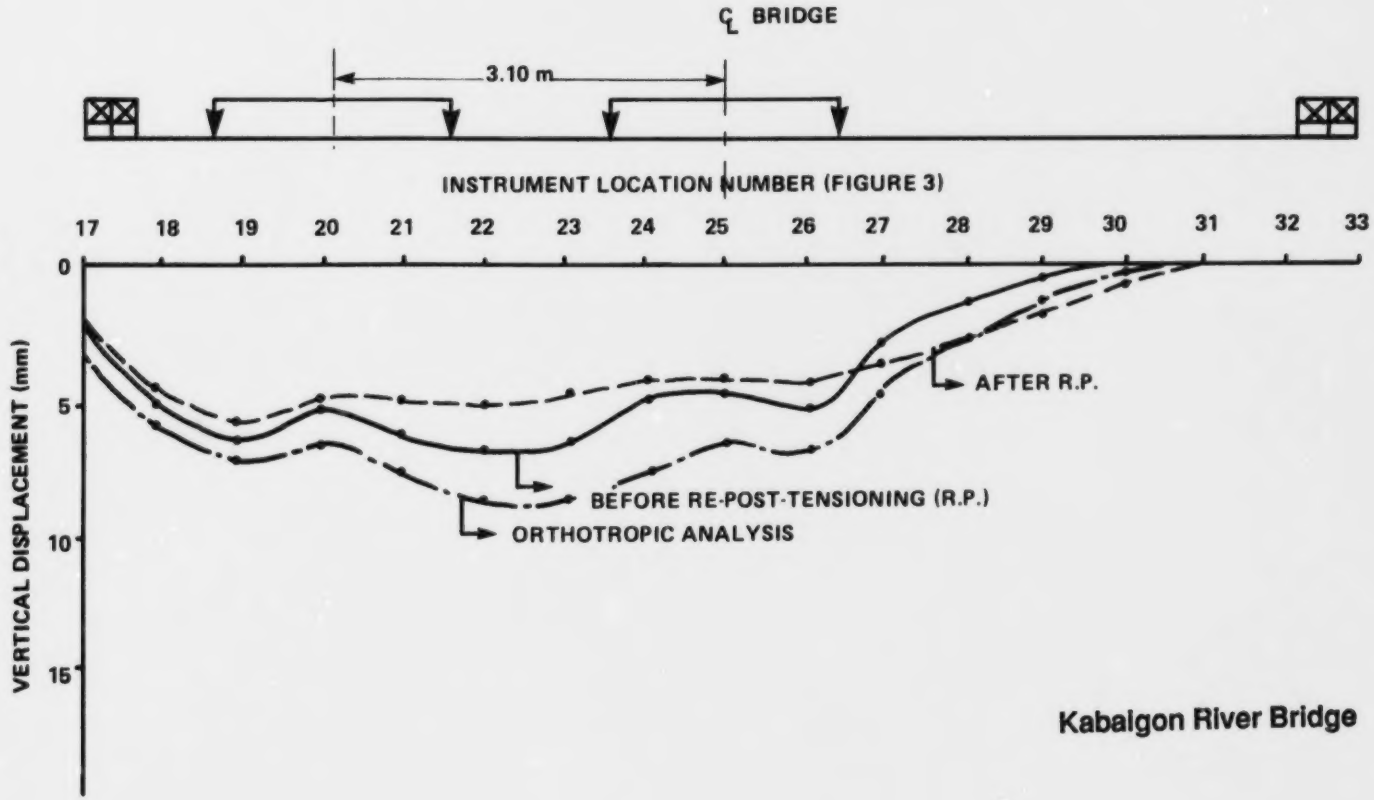


Figure 10/ Midspan Deflections, Span No. 1, Load Level 8

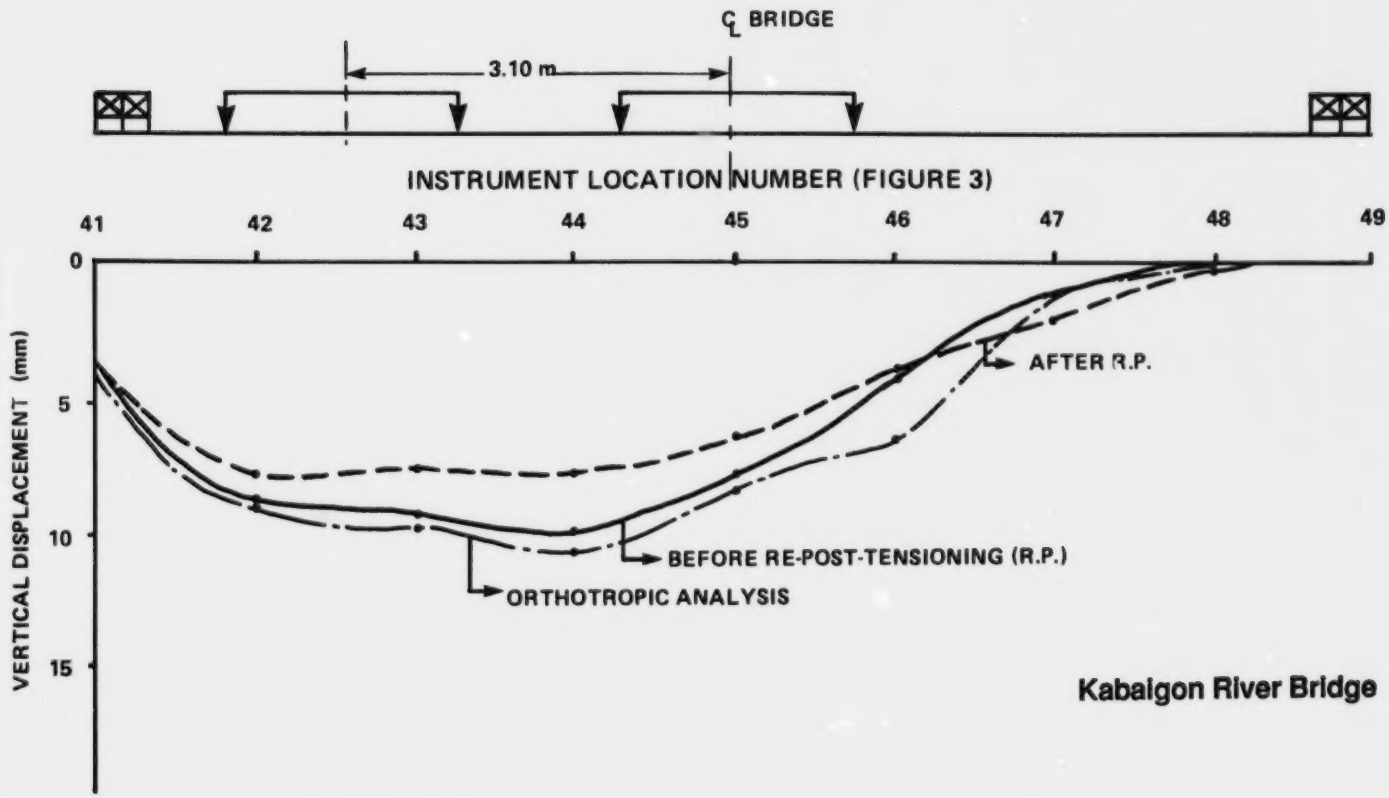


Figure 11/ Midspan Deflections, Span No. 2, Load Level 8

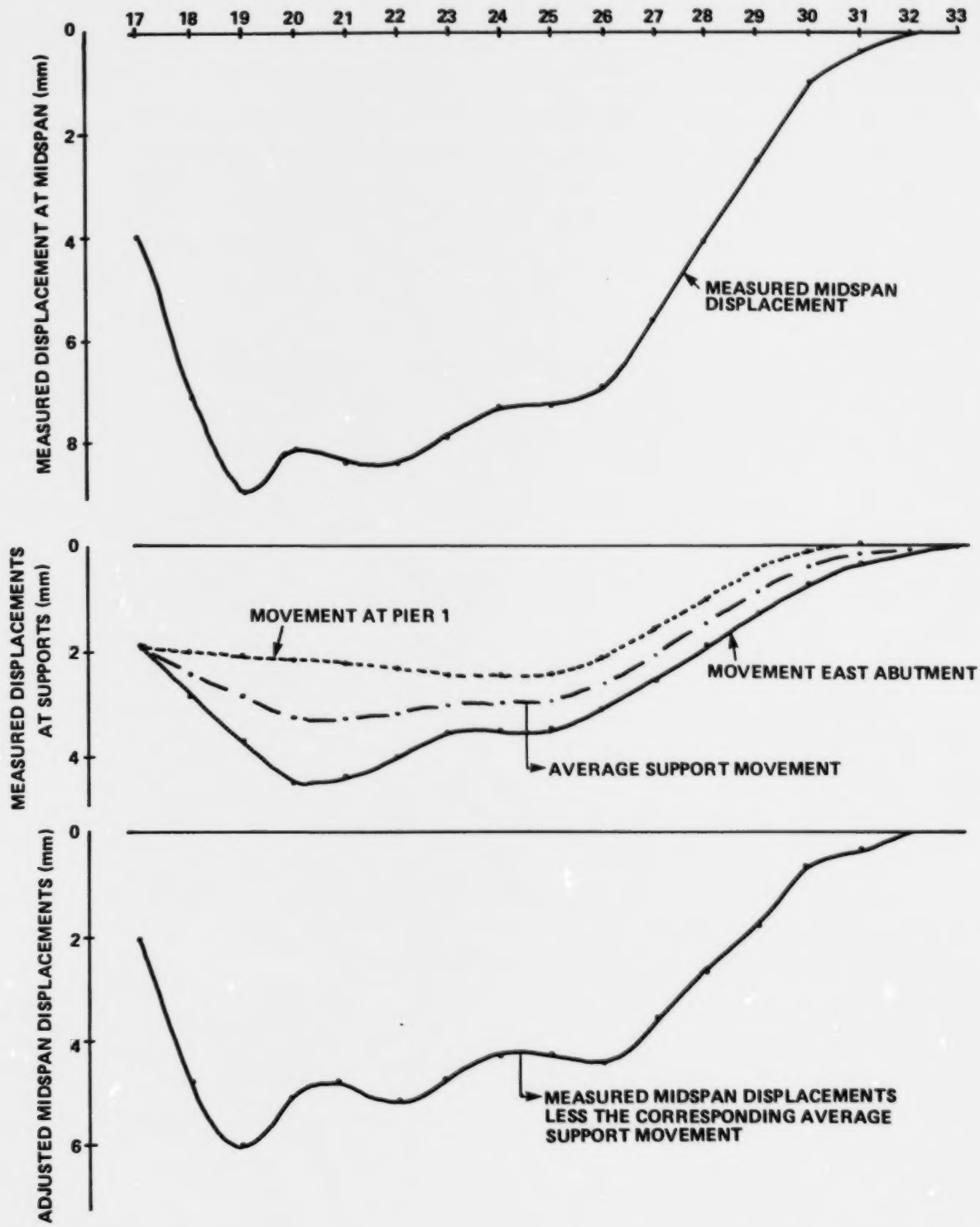


Figure 12/ Method of Adjusting for Support Movements

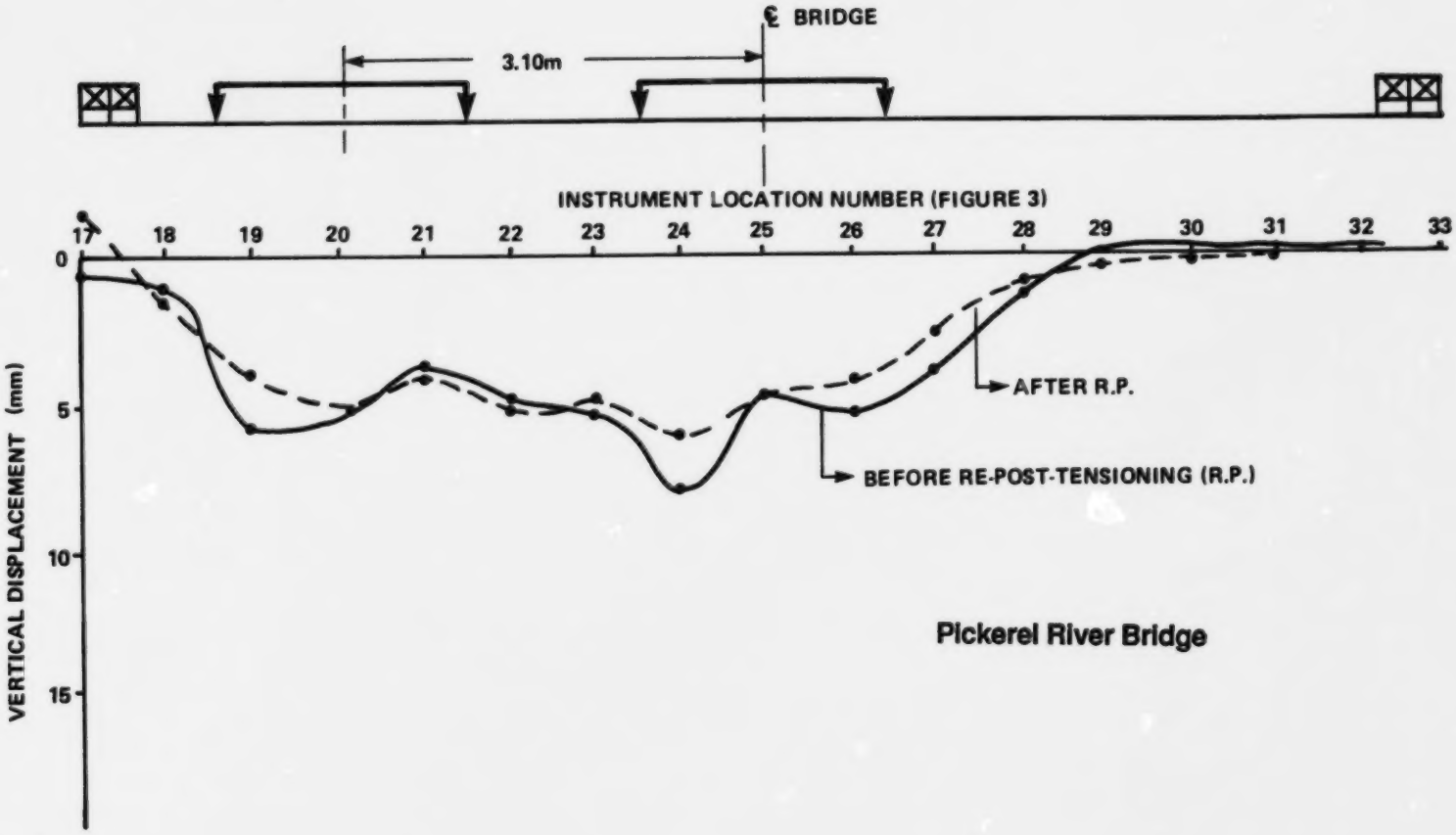


Figure 13/ Midspan Deflections, Span No. 1, Load Level 8

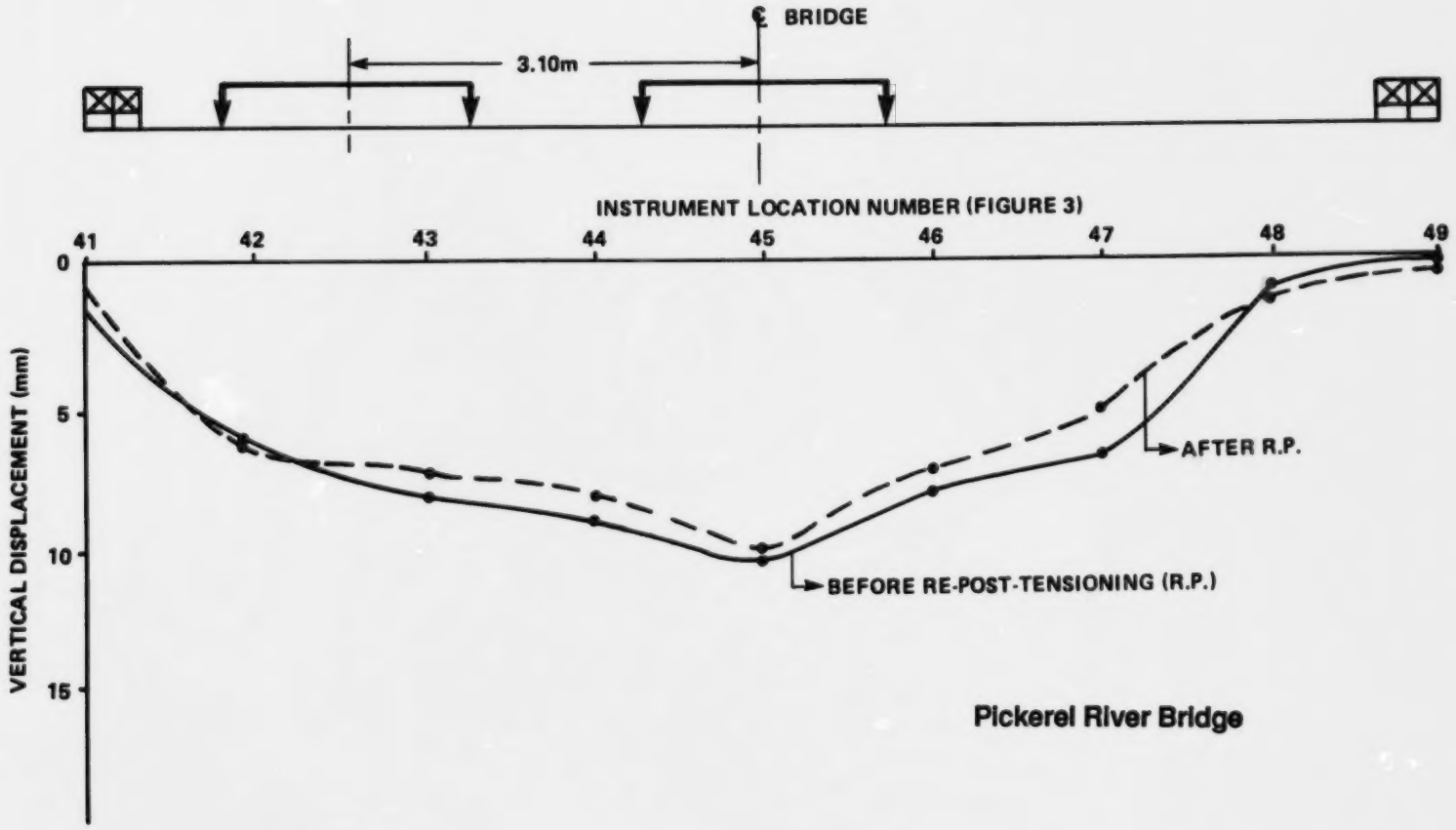


Figure 14/ Midspan Deflections, Span No. 2, Load Level 8

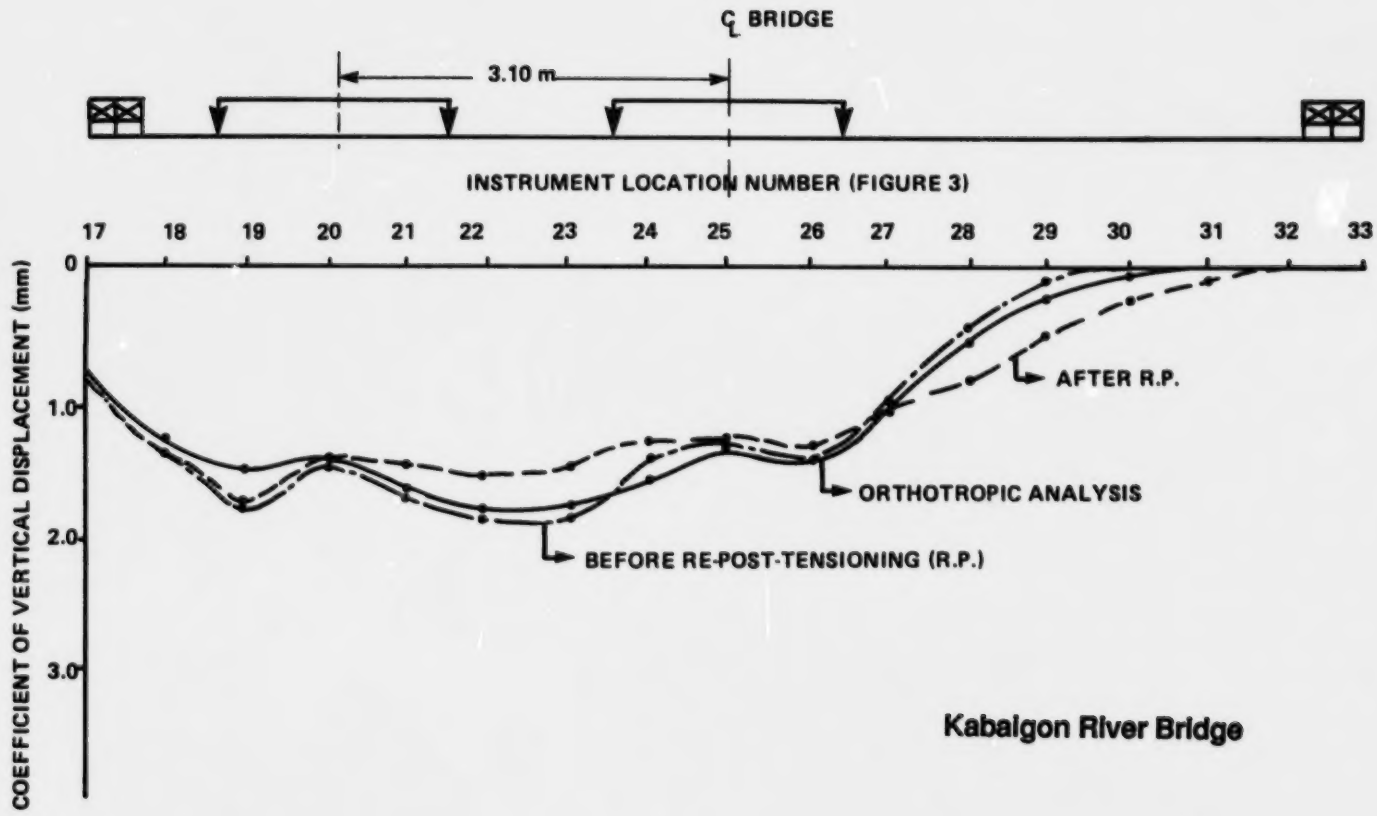


Figure 15/ Normalized Midspan Deflection, Span No. 1, Load Level 8

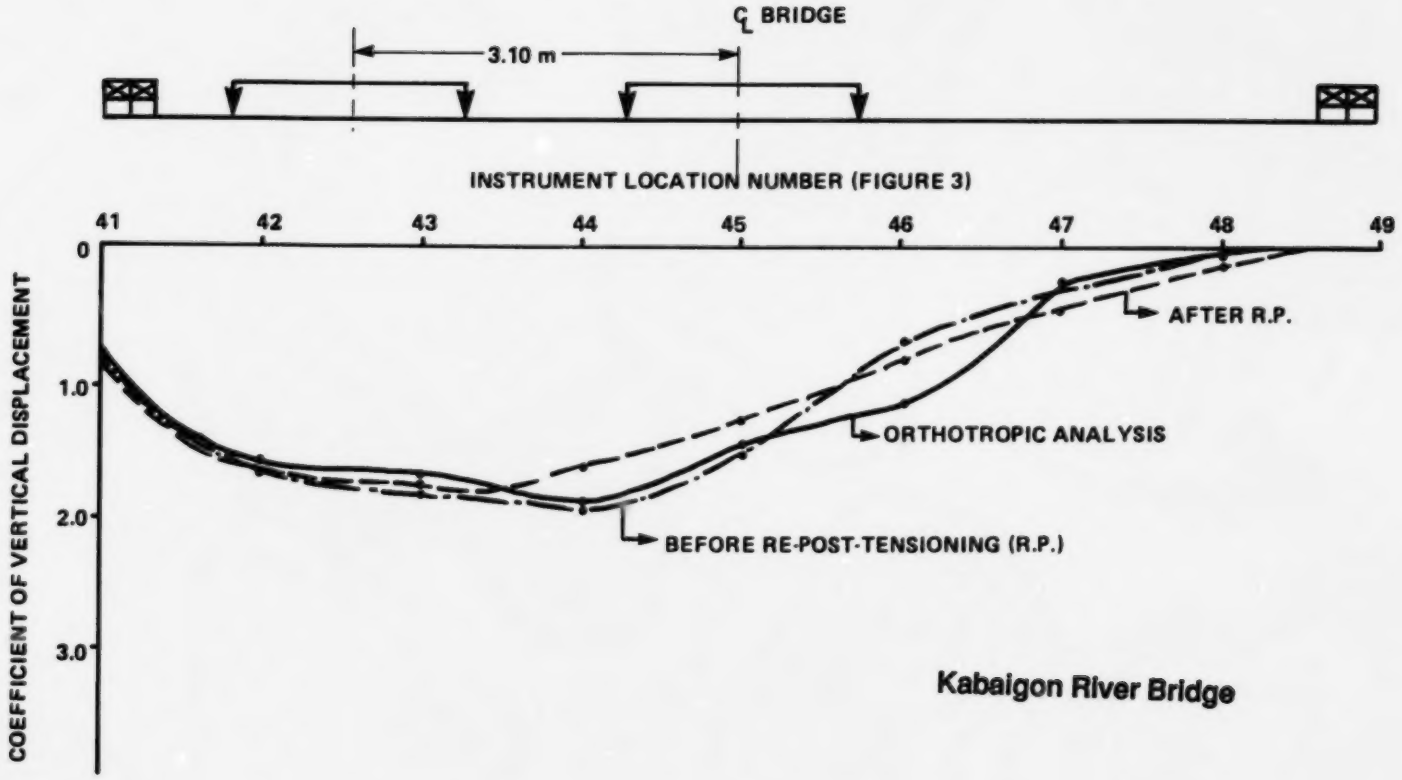


Figure 16/ Normalized Midspan Deflection, Span No. 2, Load Level 8

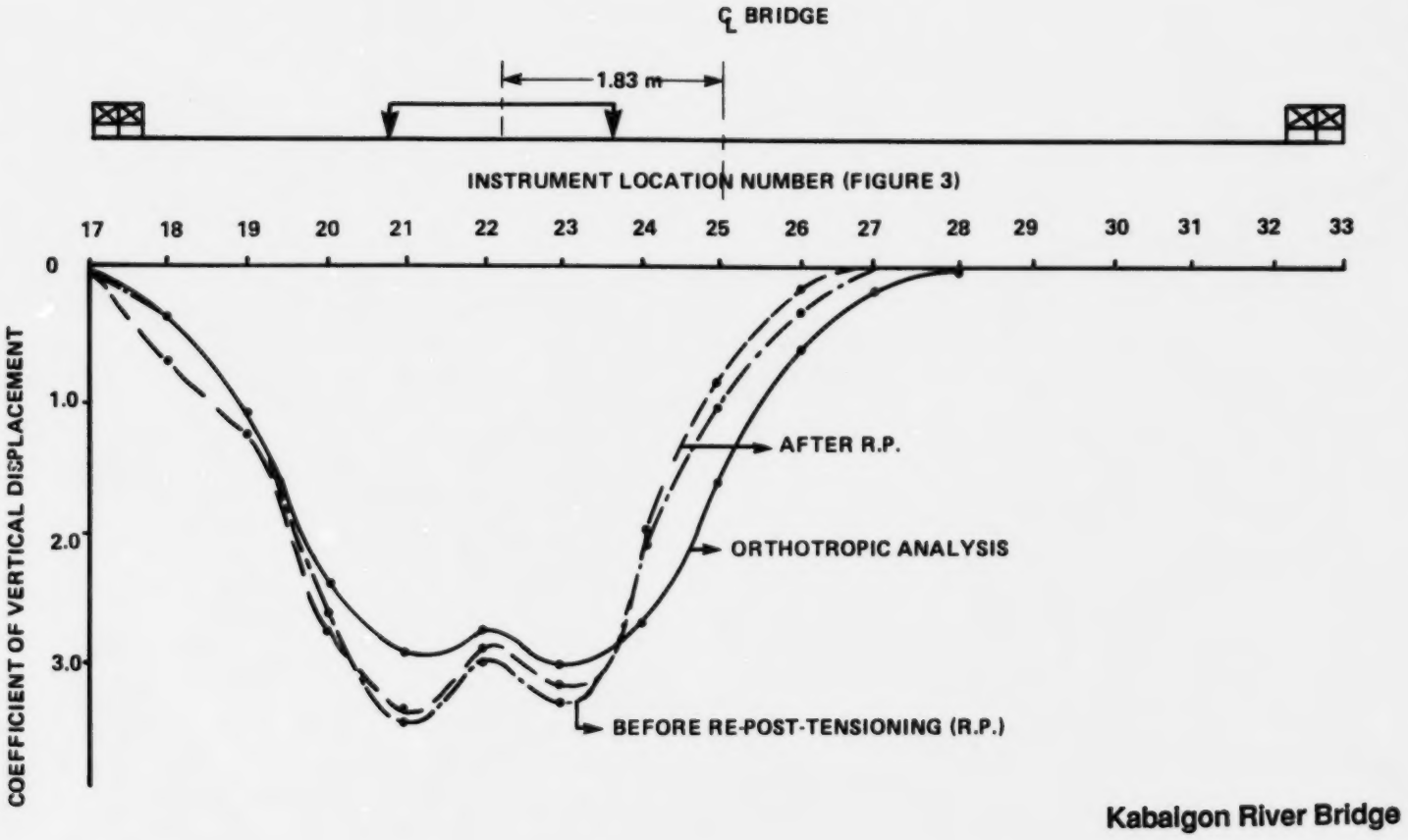


Figure 17/ Normalized Midspan Deflection, Span No. 1, Load Level 8

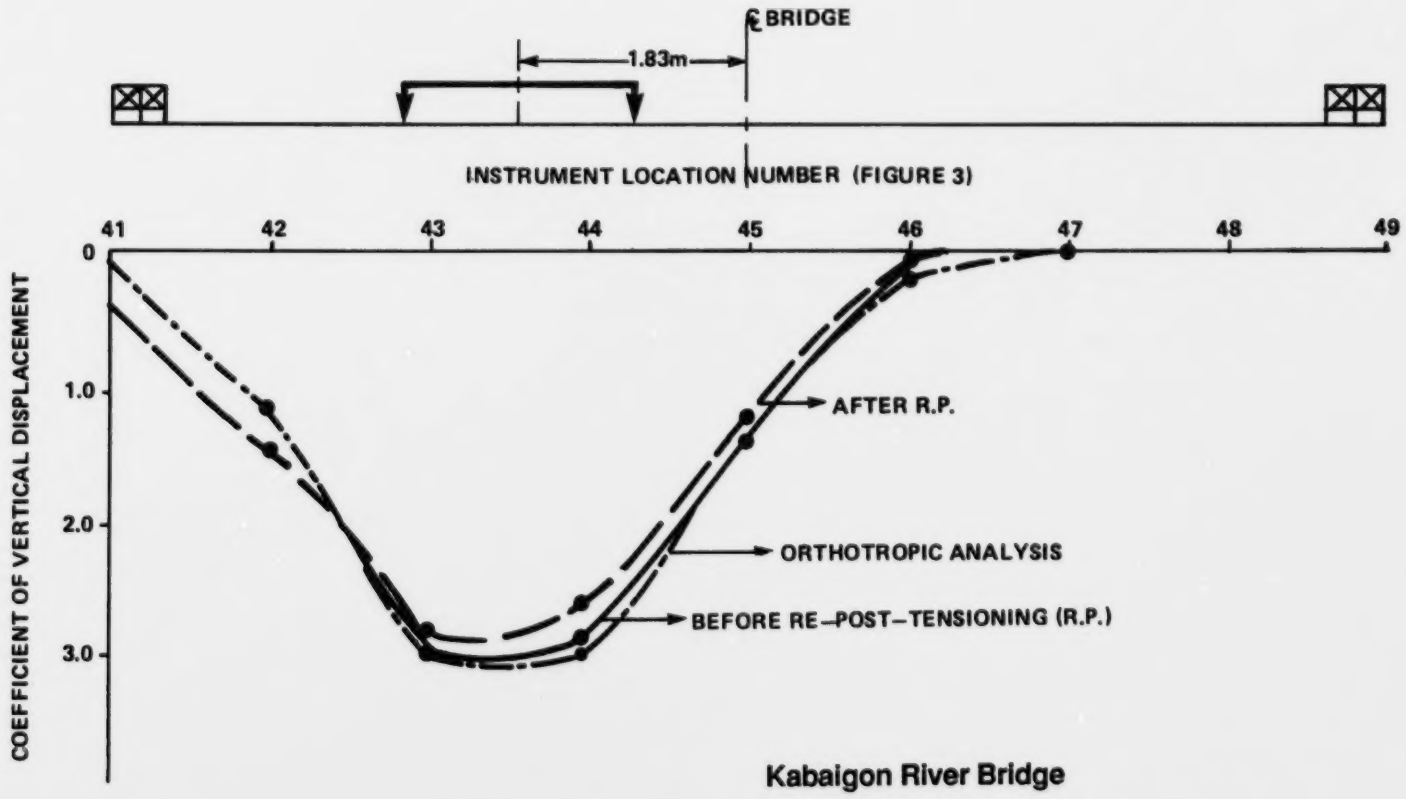


Figure 18/ Normalized Midspan Deflection, Span No. 2, Load Level 8